Restrainers at Hinges and Bearings

Introduction

Restrainers shall be installed at hinges and bearings to limit the longitudinal movement and keep the structure tied together during severe shaking of earthquakes.

The main purpose of restrainers is to prevent spans from falling off their supports during the maximum credible earthquake.

Designers are encouraged to develop new types of restrainers and should discuss their ideas with the earthquake committee. The ideal restrainer should be capable of resisting appropriate forces, restricting movements of bridge segments, dissipating energy and returning the structure segments to their relative pre-earthquake positions.

Cables and bars acting in tension do not dissipate any significant amount of energy. They store energy as they are stretched but impart it back to the structure as the segments move back together. Cable and bar restrainers may permit the ends of girders to be damaged but the damage should be repairable and not extensive enough to allow the spans to drop. Steel cables and bars acting in direct tension are probably the most economical and suitable restrainers for most bridges.

Cables tensioned repeatedly within the elastic range will store more energy than an equivalent number of steel bars of the same length. Bars will dissipate more energy than equivalent number of the same length cables if both are tensioned beyond the yield strength. However, considering all of the unknowns involved, it is not prudent to depend on a restrainer acting beyond its elastic limit.

Twenty (20) foot long ¾-inch cables will stretch approximately 4 inches at yield (39.1 kips) and 10½ inches at ultimate (53 kips). Twenty (20) foot long 1¼-inch diameter bars will stretch approximately 1 inch at yield 150 kips and about 15 inches at ultimate (188 kips).

Adequate lengths of cables or bars should be used in order to assure sufficient stretch capacity. Twenty (20) feet is suggested for preliminary purposes for new construction. The results of Equivalent Static Analysis should be carefully analyzed to assure that joint movements are kept within tolerable limits and restrainers work within the

Supersedes Memo to Designers 20-3 dated November 1989 elastic range. Excessive stretching could lead to premature column failures and could be especially critical in retrofitting structures with narrow hinge seats. Insufficient ability to stretch can cause premature failure of the restrainers.

There are not any rules of thumb for predicting whether cables are better than bars or vice versa for any particular installation. Equivalent static analysis of long and short cables and bars will give the designer information for selecting movements within a tolerable range.

Restrainers at hinges and bearings should have redundancy. There is always a chance that a single unit has a defect (due to faulty fabrication, installation, adjustment maintenance, etc.) and will fail sooner than expected. Additional units or other devices should be capable of doing their share of the job if one unit fails prematurely. Ultimate failures should be ductile. Restrainer attaching devices and details should not fail; however, when attaching short cables across joints by connecting hinge diaphragms, it is necessary to investigate the tensile capacity of the existing superstructure.

A minimum number of restrainers should be provided in all hinges of new bridges, significant widenings, and minor widenings of bridges which contain existing restrainers. Refer to *Bridge Design Specifications*, Article 3.21, "Seismic Forces" and the related BDS commentary for analysis and design procedures.

Many older bridges have seismically deficient columns as well as inadequate bearings. The deficiencies may be due to an insufficient amount of longitudinal reinforcement; too few, too small or improperly detailed ties or spirals; improperly located lap splices; or inadequate anchorage or confinement of longitudinal steel in footings or caps. These deficiencies are much more critical for structures with single columns than those with multi-column bents. The problem may be so extensive in some that replacement must be considered in lieu of strengthening. If it is obvious that deficient columns will not be retrofitted, restrainer forces must be limited to the force required to fail the columns. However, the designer must assure that the total support system will be stable, regardless of the severity of the damage, and collapse will not occur.

New Construction

Restrainer Unit – Type 2 (Reference Bridge Standard Detail Sheet XS 12-53)

1. The designer should determine the total number of longitudinal restrainers required and the total number of cable units needed per restrainer. Consider symmetry when locating restrainers. A diagram should be detailed on the plans to show the location of restrainer units and number of cables per restrainer.

   Restrainers placed as shown in Figure 1 provide symmetry for a balanced design and one opening provides access to two restrainers if inspection or maintenance is required.

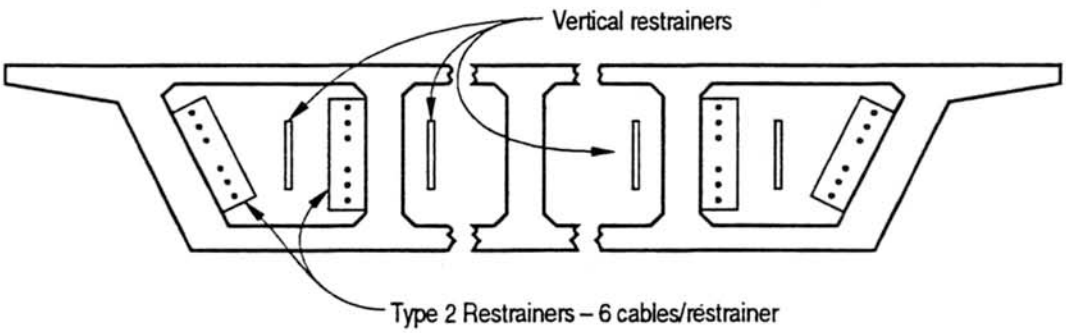


Figure 1

2. The two center cables should be spaced at 12 inch centers in order to clear the hinge reinforcement. Increase the spacing if the bearing thickness exceeds 2 inches.

   Remaining cable units should be spaced at 6 inch minimum centers. Use as many cables per restrainer as this space will allow.

3. Bearing plate length shall be determined by the Designer and noted on "DETAIL-PLATE B" on standard sheets.

4. The anchor nuts should be snugged to take slack out of the cables, then tightened one turn per 5 feet of restrainer length. The nut should then be backed off a distance equal to the maximum additional amount that the hinge joint is expected to open due to temperature, prestress shortening, and shrinkage and locked with a jam nut.

5. Consideration should be given to the opening of hinges in prestressed spans due to shortening of the spans caused by prestressing. Restrainers for new prestressed box girder bridges should be anchored after the prestressing operation has been completed. Sufficient thermal information should be supplied on the plans, based on a given timeframe, which will allow the Structure Field Representative to dictate the anchor gap. This gap should be shown on the plans similar to dimensions used for joint seal movements.

6. Restrainer ducts, continuous across hinge construction openings in prestressed bridges, should include a slip-type expansion device to prevent a tension fracture due to prestress shortening.

7. Access openings should be provided for the anchorage at both ends of the restrainer. Permanent soffit openings are recommended. Specifications for cleaning formwork from cells should be modified when the permanent openings are for restrainer access only. Cleanout should be limited to the restrainer access corridor.

8. For transverse seismic of single column or long, slender, multicolumn bents, consider added diaphragms for torsion rigidity and girder side reinforcement for deflection problems.

9. For situations where movement must be held to a minimum, high strength rods may be substituted for the ¾-inch diameter cables. Also, for cases where access holes need to be held to a minimum, construction procedures using rods may be beneficial.

Restrainer Unit – Type 5

Structures consisting of precast girders have available restrainer details on Bridge Standard Detail Sheet XS 12-56.

Vertical Restrainers

Use vertical restrainers when uplift forces exceed 50 percent of the DL reaction. (Refer to *Bridge Design Specifications* 3.21.6). See Bridge Standard Detail Sheet XS-12-53.

Retrofitting

The San Fernando earthquake of February 9, 1971, and the Loma Prieta earthquake of October 17, 1989 were the only events to cause any significant amount of damage to any of California's bridges. The total amount of earthquake damage to bridges experienced before that time was minor and was generally ignored. Five small earthquakes since 1971 have caused some minor damage and the collapse of two spans of one four span bridge. The knowledge gained by studying that damage gives an insight into how structures react to seismic shaking and what can be done to mitigate the damage expected from the larger earthquakes that are certain to occur in the future.

Studying the damage from minor earthquakes is valuable because it demonstrates two stages of failure and it is not necessary to speculate on the sequence of events as might be done when conducting a post-mortem on a completely collapsed structure. Although there is a wide variety of bridge details used in California and in other countries, there is a consistency in the seismic damage experienced.

Even though bridges can be retrofitted to increase their resistance to total collapse in the event of a major earthquake, they will still experience minor damage from smaller seismic events.

Initial retrofitting completed, consists of tying units of the superstructure together and to their supports. Although this directly solves only the problem of the spans dropping off of their supports, it partially alleviates some of the other seismic deficiencies. The column retrofit program began in 1986, and consists of developing techniques to improve the ductility performance of bridge substructure components. The program is currently in both design and construction phases.

Retrofitting Philosophy

- The goal of retrofitting is to increase the seismic resistance of a bridge to minimize the probability of total collapse.
- Retrofitting should eliminate or reduce the hazard of loss to human life.
- If practical, retrofitted bridges on critical routes should be able to carry emergency vehicles after a major event.
- It may *not be practical or economically feasible* to retrofit a bridge so that it will have the same seismic resistance as a new structure designed to current specifications.
- Retrofitting is generally not recommended if the only expected deformation is a small probable maximum vertical displacement (≤ 6 inches) and emergency

traffic can be accommodated by ramping the vertical offset with dirt or other readily available material until permanent repairs can be made.

- Main spans of pedestrian overcrossings that could drop on vehicular traffic should be retrofitted. Other spans need not be retrofitted unless they can be done at a low cost when the main spans are done or unless there is a considerable amount of school or other high volume pedestrian traffic that could be injured.
- Appearance of structures being retrofitted should be given consideration. Generally speaking:
  1. If the cables are between girders, above the girder bottom flange, or are attached by means of small fittings they are *least* objectionable.
  2. If the cables are wound around columns or other structural members visible from a position outside the structure they are *more* objectionable.
  3. If the cables are visible in silhouette and are obviously not a part of the major structural scheme, they are *most* objectionable.
- All of the above is further influenced by the environment: sky, background, color, character, etc.

It might be argued that precast or steel beam structural types fall into a group titled "articulated" and might be expected to contain hardware. This hardware should be minimized in size and prominence to retain its place in the general structural order.

General tidiness in detailing, a little paint, avoiding profile view contamination by the "system" and utilizing clips or secondary fasteners in lieu of cables everywhere, could help preserve the appearance of structures retrofitted by narrow minded efforts of "restrain at all costs."

Structures Maintenance is concerned about reduced clearances beneath steel structures due to added features when earthquake retrofitting. The primary feature which has reduced available clearances, and subsequently causes problems when issuing oversize load permits, is the restrainer bracket mounted on the underside of the lower girder flange over traveled ways including shoulders. Features attached to bent caps may reduce horizontal clearances also.

Designers should be aware that anytime bridge modifications are processed, resulting impairments to both vertical and horizontal clearances of the traveled way, including shoulders, should be kept to a minimum. Reductions of vertical and horizontal clearance to less than existing clearances of adjacent structures should be avoided when possible. When not possible, Structures Maintenance Permit Section should be advised of intended details. A bracket with outriggers mounted to the outside of bottom girder flanges is one method of avoiding reduced vertical clearance.

Considerations for Retrofitting

It is not possible to formulate simple rules to determine whether or not a structure requires retrofitting to improve its performance during an earthquake or, if so, what type of retrofitting it requires. In addition to the geological and seismological conditions at a particular site, any one of a combination of two or more of the following physical features of a bridge could determine whether or not retrofitting is advisable:

- Type of construction
- Physical condition of the bridge
- Length
- Width
- Ratio of length to width
- Skew
- Curvature
- Number and location of joints
- Type of bearings and hinges
- Abutment type and height
- Number of spans
- Restraining devices (shear blocks, curtain walls, etc.)
- Type and degree of failure anticipated if not retrofitted
- Column reinforcement details
- Lifeline requirements
- Sociological considerations
- Utilities carried

The following figures illustrate some of the different conditions that should be considered in determining whether or not a bridge should be retrofitted and the type of retrofitting that is required. It should be remembered that adverse geological conditions may complicate many of these situations.

Figures 2 and 3. As a rule, single span square structures should not require retrofitting. Although they may sustain some damage, they should be serviceable unless they cross a fault. If they do cross a fault, it is not likely that retrofitting will be effective.

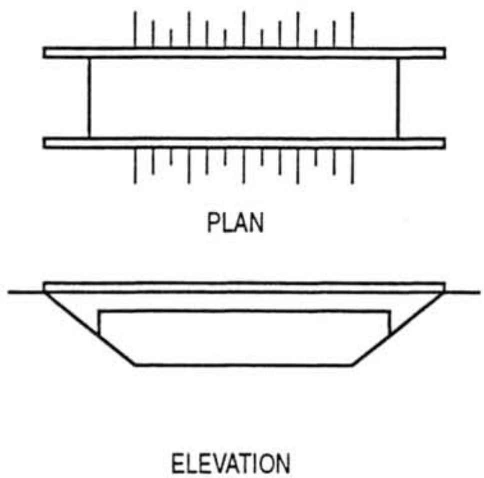


Figure 2

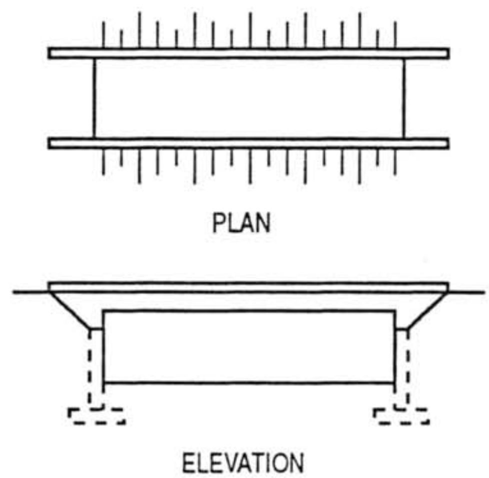


Figure 3

Figure 4. Skewed bridge spans have a natural tendency to rotate even when not shaken. Longitudinal seismic shaking produces transverse components of force which tend to rotate the span each time it moves back and forth. Transverse seismic forces cause one end of the span to bear against one abutment while the opposite end tends to swing free — in the natural direction of rotation. If the bearings, curtain walls, or other means of restraining rotation fail, the span can rotate excessively. In some cases the span may drop only a few inches and the bridge can be used with minor inconvenience and easily restored to its pre-seismic condition. If the supporting seats are very narrow, the span can drop and the bridge will be a total loss.

Figure 5. If a bridge is very wide in relation to its length, it may be locked between its abutments so that the rotation described in Figure 4 is negligible. Longitudinal shaking may cause insignificant damage. Transverse shaking may damage the bearings, shear keys or curtain walls, but there is much less probability for the more serious damage that might be expected with a longer, narrower structure.

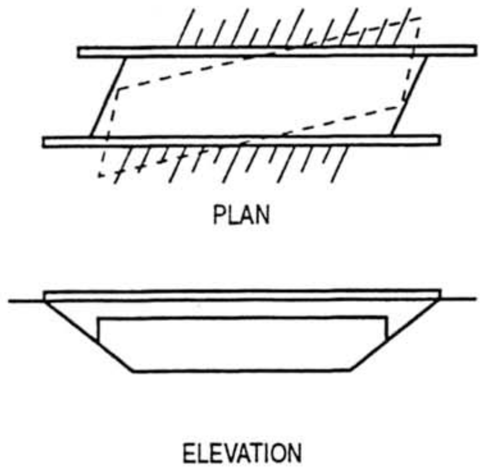


Figure 4

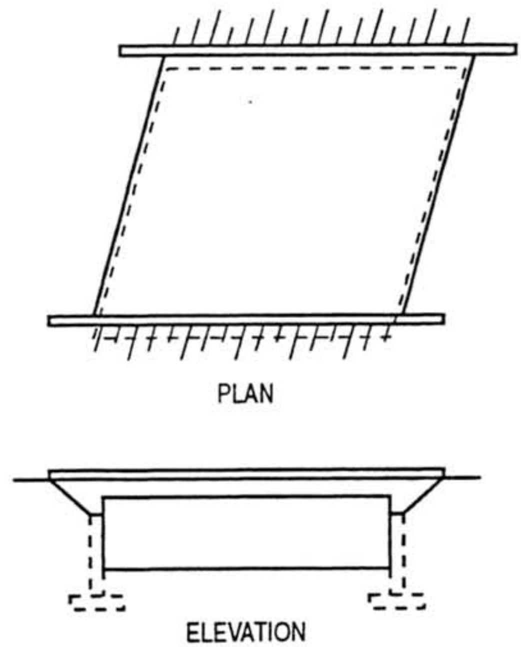


Figure 5

Figure 6. Long, non-skewed, continuous bridges with diaphragm type abutments and without intermediate hinges or joints need not be retrofitted. Bridges with bearings at the abutments may require transverse restrainers at the abutments if it is determined that there is insufficient restraint provided by bearings, curtain walls, shear keys or other restraining features.

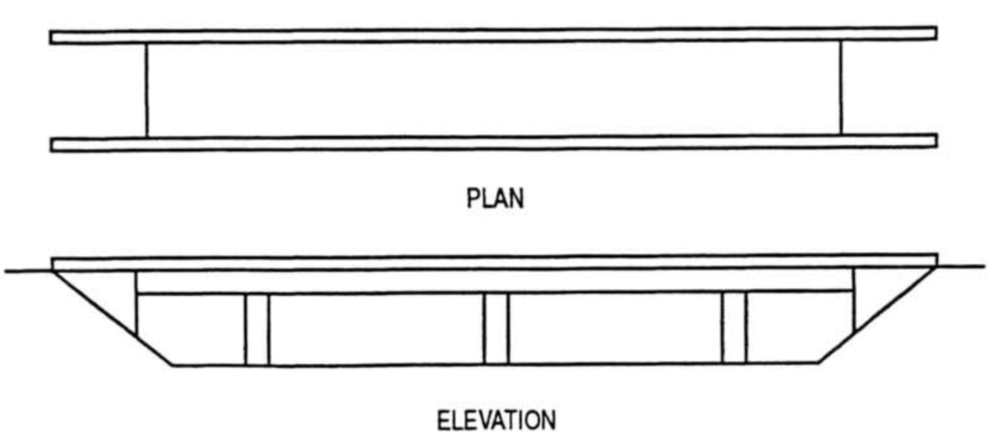


Figure 6

Figures 7 and 8. Long, continuous, skewed or curved bridges without intermediate hinges or joints are more prone to seismic damage than similar square bridges. Due to the nature of the details, they will probably require additional transverse restraint at the abutments for a lower seismic level of shaking than a similar square bridge.

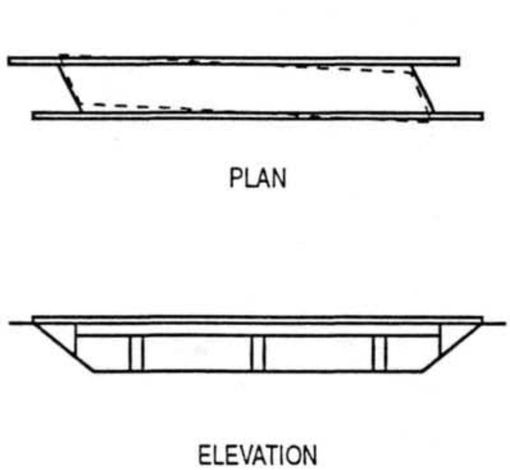


Figure 7

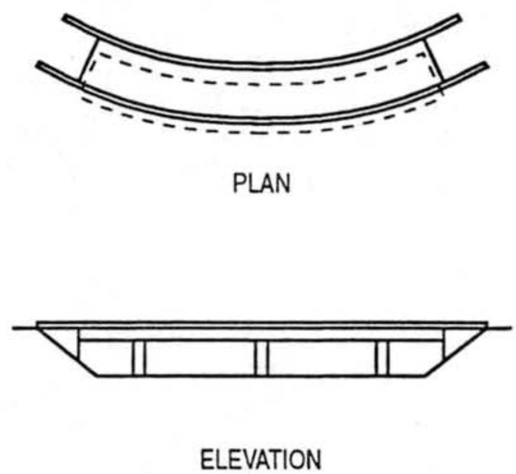


Figure 8

Figure 9. Segments of a superstructure which aren't adequately restrained act independently and may tend to separate when shaken. If the bearings or other means of transverse restraint fail, longitudinal restrainers (if installed) may act as tension members in a large horizontal beam. Restrainers should generally be placed as close to the edge of the structure as possible so they can offer the maximum amount of resistance for this condition.

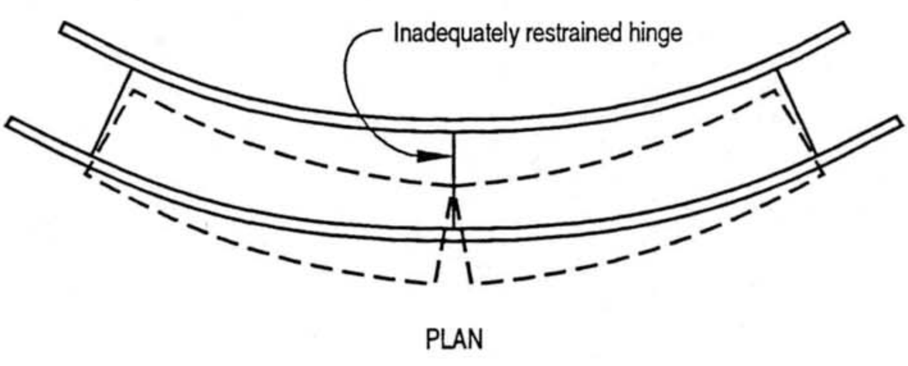


Figure 9

Figure 10. Sharply curved bridges which have seismically inadequate bearings at an abutment and very flexible or seismically deficient columns may require additional restraint at the abutments. Abutment restraint, in cases such as this, may alleviate some column weaknesses. One common problem, however, is that the abutment may not be capable of resisting the anticipated forces. Many abutments are very light-weight and the shear resistance of the soil or piles may be insufficient.

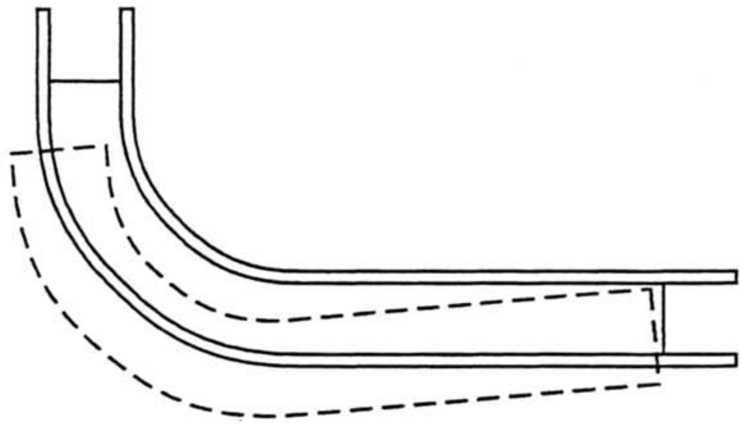


Figure 10

Figure 11. Long continuous reinforced concrete slab bridges as a general rule, need not be retrofitted with hinge restrainers. This is based on the assumption that if the suspended span becomes unseated the deadload of the resulting cantilever will not be sufficient to make it fail. Long span non-standard slabs should be checked for this criteria. Retrofitting should be required if there are two hinges in the same span or if the unseating of any hinge will lead to a dropping of any or all spans with dead load only. It is assumed that those unsupported ends will be quickly recognized and reseated, temporarily strutted, or traffic barricaded from using the bridge before any serious accident occurs.

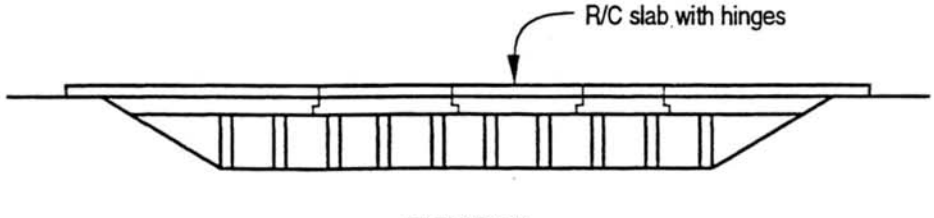


Figure 11

Figure 12. Any bridge with 6 inch or 8 inch steel angle hinges, or equivalent, should be retrofitted regardless of what seismic area it is in. Due to shrinkage, seasonal variations, and other factors, many of these hinges have marginal seating length under even normal conditions. Any seismic shaking could cause them to become unseated.

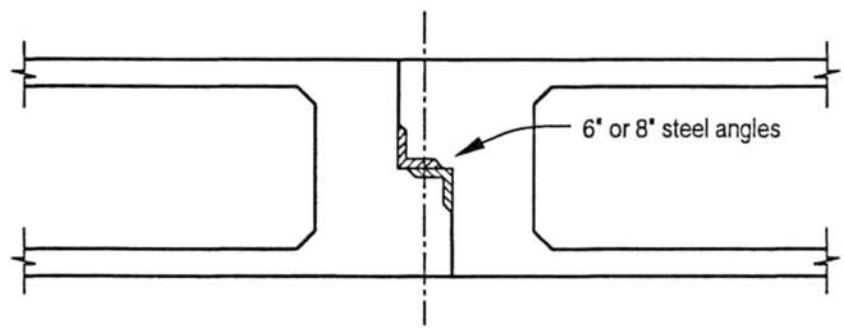
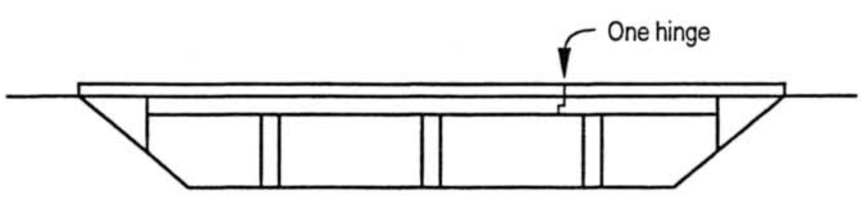


Figure 12

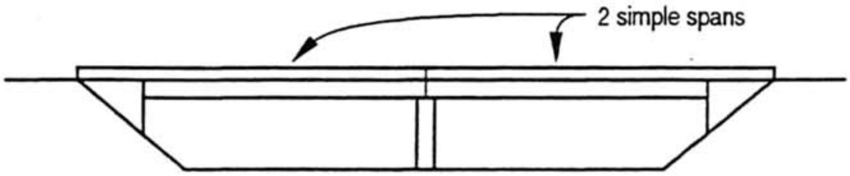
Figures 13 and 14. A non-skewed, straight, continuous bridge with only one hinge or a nonskewed straight bridge with two simple spans may be designed for the minimum of 40 percent of the dead load of the lighter segment of superstructure connected. This would be consistent with the rough assumption made for the resistance and action of the earth behind the abutments.

The influence of the earth behind abutments becomes relatively less important if the superstructure is curved or skewed. The equivalent static force method or other approved analysis should be used for designing restrainers for these structures if they are skewed or curved.



ELEVATION

Figure 13

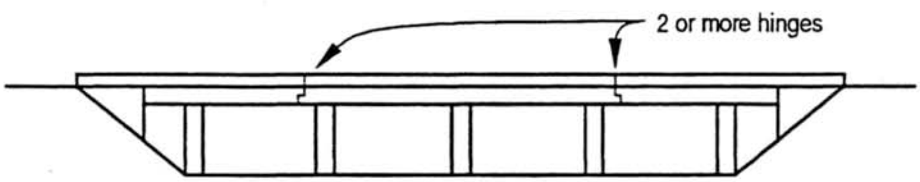


ELEVATION

Figure 14

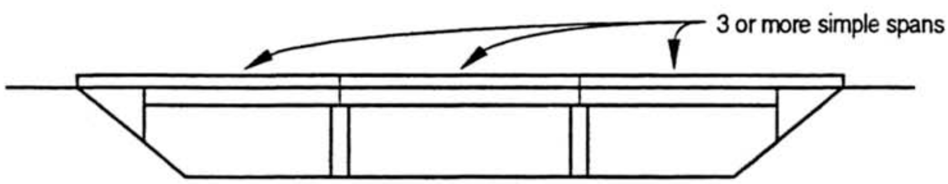
Figure 15. A dynamic analysis for column and abutment forces should be considered for any bridge with two or more hinges.

Figure 16. For multiple simple spans, a dynamic analysis is not recommended. A static analysis of each bent is the recommended procedure.



ELEVATION

Figure 15



ELEVATION

Figure 16

Figure 17. Connecting the ends of girders together in adjacent spans may be satisfactory for short structures with only a few spans and wide bent caps where it seems certain that the ends of girders won't drop off the bents. This detail can also be used where it is considered that the additional longitudinal forces produced by connecting the girders to the bent caps (see Figure 18) may fail the columns. Although the bearings will probably fail, the superstructure will not fall very far and the bridge will not be completely out of service.

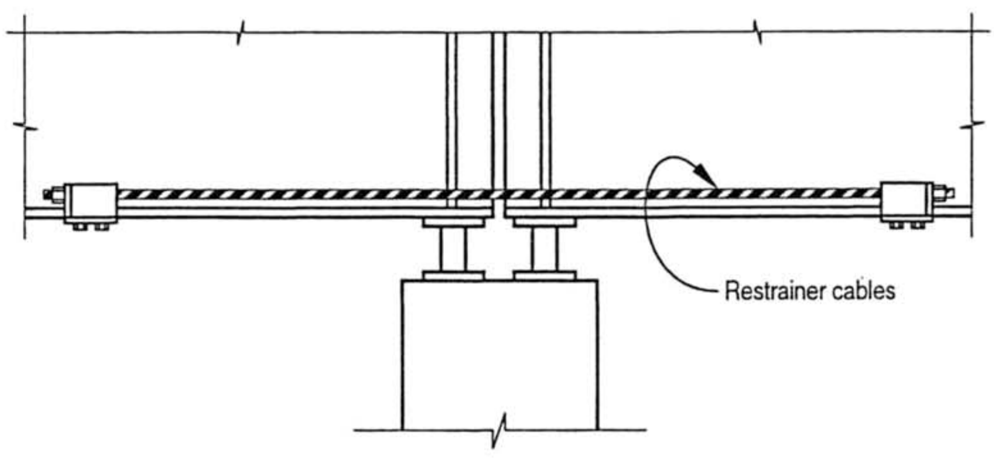


Figure 17

Figure 18. This detail is generally preferred to the one illustrated on Figure 17 with the spans butting against each other. The restrainers must be able to resist the force produced by both spans supported on that pier and possibly adjacent spans as well. Generally, not more than two frames or two simple spans should be considered to cause demands on restrainers from either side of a joint. Vertical clearances under the structure should be considered.

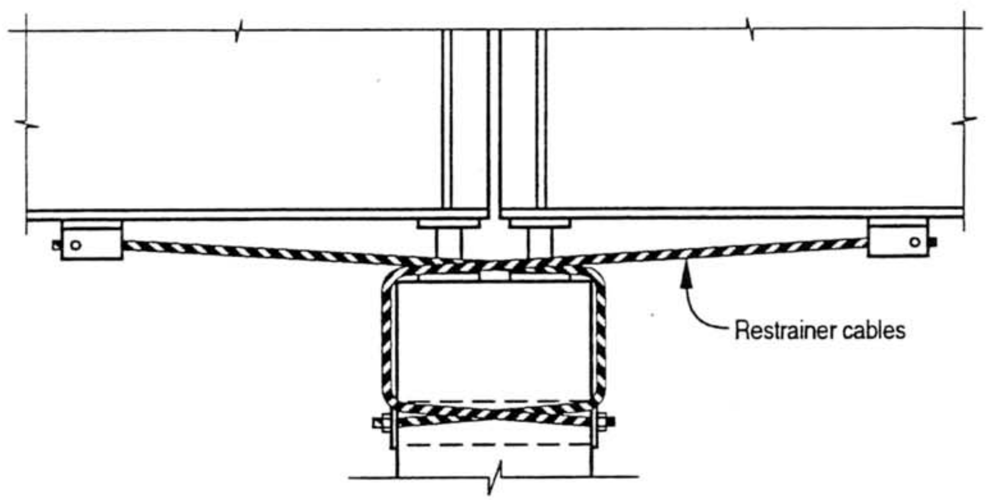


Figure 18

Figure 19. Suspended spans are particularly vulnerable to seismic shaking. Curved and skewed alignments greatly increase their vulnerability.

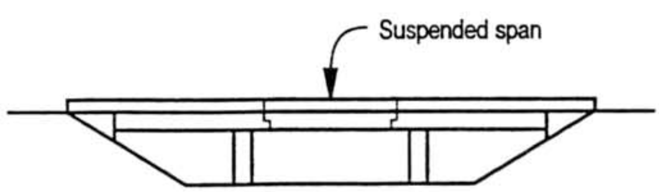


Figure 19

Figure 20. It can generally be assumed that any seat type hinge used with steel girders will need additional transverse, longitudinal, and vertical restraint in even moderately severe seismic areas.

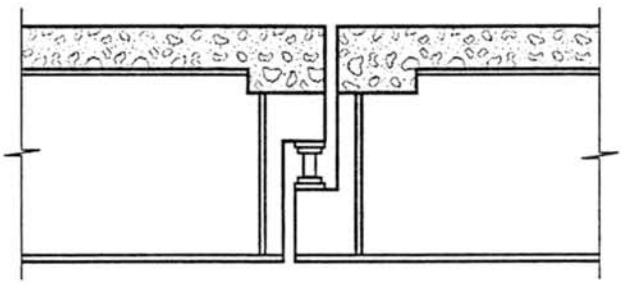


Figure 20

Figure 21. Hanger type hinges generally have more seismic resistance than the seat type shown in Figure 20, but are still subject to seismic damage. These hinges often have steel bars or angles that bear against the opposite web, or lugs attached to the flanges, which were designed to keep the girders aligned transversely for wind forces. Those devises are usually structurally inadequate and are too short to be effective with even moderate seismic shaking. Consideration should be given to replacing them or adding supplemental transverse restrainers.

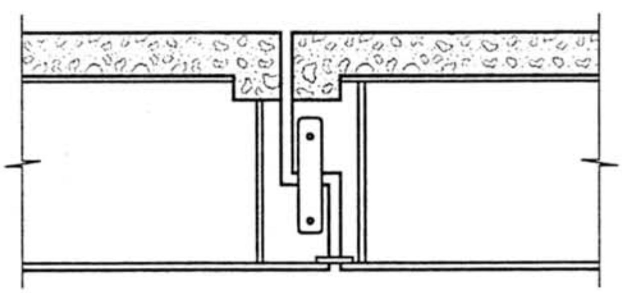


Figure 21

Figure 22. Very few older bridges have bearings that will not fail in a moderate or greater earthquake. It should be anticipated that a bridge superstructure can be displaced transversely. If the exterior girder of a multi-girder bridge is moved beyond the end of a bent, it is likely that the side of the bridge may be severely damaged and the use of a shoulder or lane will be lost, but traffic can be routed over a portion of the bridge with few or no emergency repairs. This is considered to be an acceptable risk.

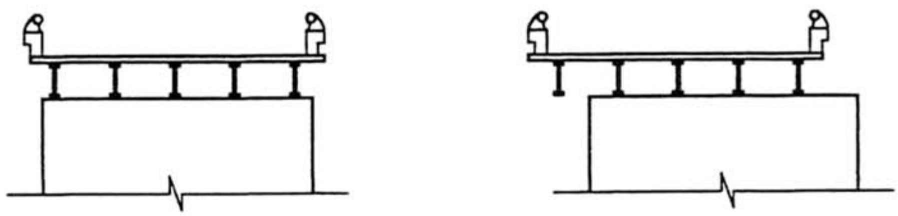


Figure 22

Figure 23. If the superstructure of a two or three girder bridge is displaced transversely so that one line of girders loses its support, the entire bridge may collapse. Adequate transverse restraint should be provided.

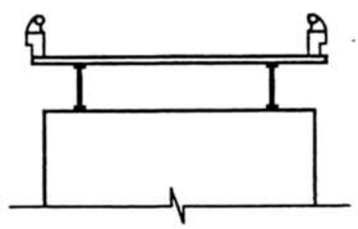


Figure 23

Figure 24. In most locations it is generally not practical to restrain longitudinally the superstructure at an abutment. Supplemental supports or pedestals may be provided to prevent the superstructure from dropping by extending the seat length and by acting as transverse shear keys. This same principle may also be applied at bents in certain circumstances.

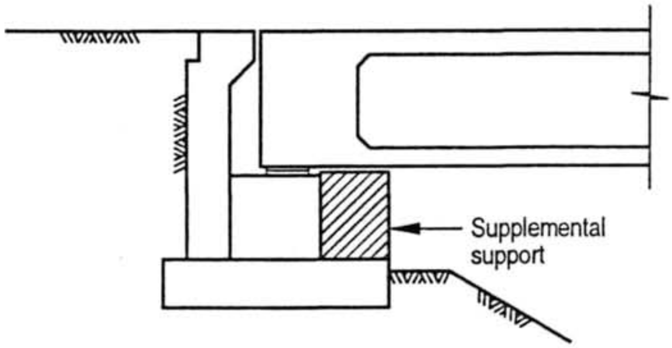


Figure 24

Figure 25. Numerous types of steel bearings used on various types of steel and concrete bridges have been damaged by relatively minor seismic shaking. It should be assumed that they will fail in areas where the maximum credible bedrock acceleration is 0.3g or greater. If the failure of any type of bearing will result in the superstructure dropping 6 inches or more without falling off of the pier or abutment, consider replacing the bearings with modern type or adding bolsters that will minimize the drop.

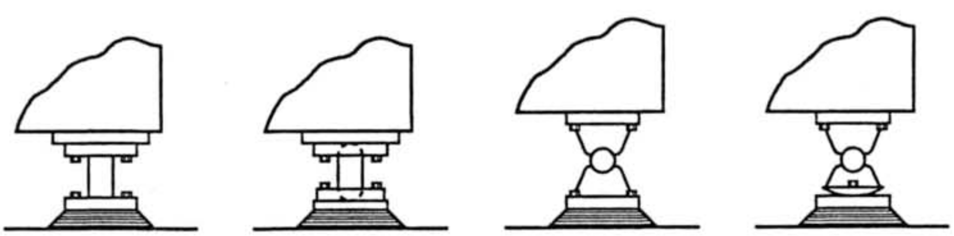
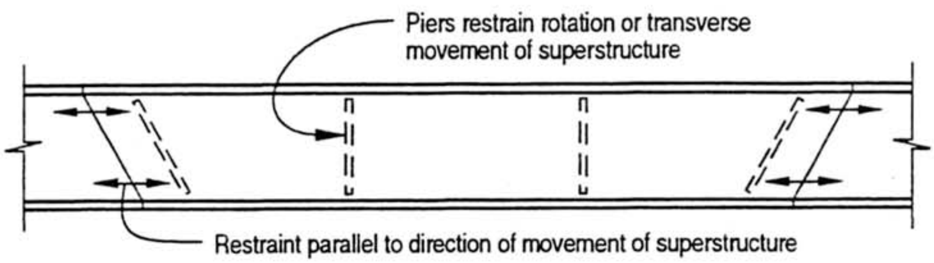


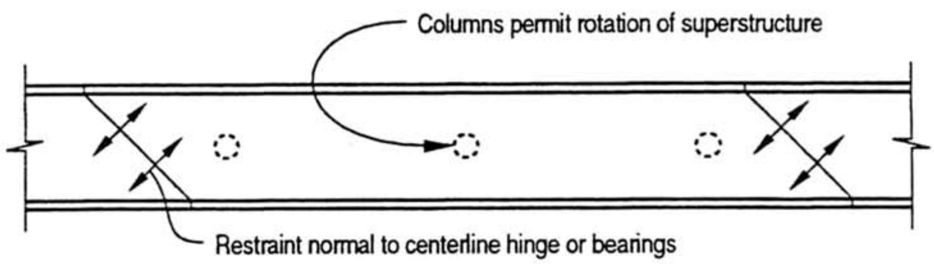
Figure 25

Figures 26 and 27. The rigidity of piers and bents can control the direction of movement of a structure. Restrainers will be more effective if they are oriented in the principal direction of movement.



PLAN

Figure 26



PLAN

Figure 27

Design Considerations

Bridge bearings have historically been one of the most seismically sensitive bridge details and their ability to resist earthquake forces cannot be evaluated with any great degree of preciseness. They are often the primary cause for many complete seismic failures. There are many cases where they were damaged by minor quakes and would undoubtedly have permitted spans to drop if the shaking had been a little stronger or lasted a little longer.

As a general rule, a designer should be very cautious about assuming that bridge bearing anchor bolts, keeper bar bolts or welds, and similar details have any significant effect in keeping a bridge superstructure on its supports during a major earthquake. The following shortcomings of bridge bearings should be considered:

1. All of the bearings at the end of a span more often than not do not resist horizontal forces simultaneously. Because keepers or other devices are not set with exactly the same clearances, only one half, or fewer than one half, of the bearings will initially resist a horizontal force in one direction. It is not uncommon for bearings at one end of a span to be damaged to varying degrees after an earthquake.

2. Grout pads under bearing masonry plates have traditionally given trouble during and after construction and have been one of the main sources of trouble in minor quakes. Failure of a grout pad will allow the bearing assembly to move and subject the anchor bolts to combined bending and shear.

3. The common detail of a girder seated on an elastomeric pad will subject anchor bolts to combined bending and shear.

4. Anchor bolts are frequently threaded below the top surface of the pier or abutment seat. This gives a reduced area for shear and minimal resistance to bending before failure occurs due to notch sensitiveness at the root of thread.

5. Although it is less common, some anchor bolts are too close to the edge of the bearing seat and will spall off the concrete when subjected to horizontal loads.

6. Keeper bars allow movement between the sole plate and bearing bar or rocker. Sliding takes place on this surface. Sliding obviously does not start until the horizontal force exceeds the vertical load times the coefficient of friction. When this happens, does it result in an impact on the keeper or anchor bolts? If so, it can increase the calculated force considerably.

7. Bridge bearings may not be what they are represented to be on "As Builts" plans or maintenance records. Adjustments to keepers or other details are occasionally made after construction is completed and the details or workmanship may be inferior to the original.

Details

Three quarter inch cables and 1¼ inch high strength bars are both considered suitable for retrofitting. Since bars stretch considerably less than the equivalent number of the same length cables acting within the elastic limit, and consequently store less energy, longer lengths or greater resisting capacities may be required. Since bars are more readily sheared by transverse movements of hinges, additional transverse restraining devices may be required — such as solid steel bars or concrete filled steel pipes.

During the course of the retrofitting program a number of details have been used and questioned for various reasons. The following decisions have been made in conjunction with concerned parties and should be followed in future contracts:

1. Steel deck cover plates in District 7 should be galvanized. In District 4 they should have epoxy and grit on the riding surface. Although other districts have not expressed preferences, use District 7's policy in Districts 5, 8, 9, and 11. Use District 4's policy in Districts 1, 2, 3, 6, and 10.

2. Compressed closed cell neoprene has been used to prevent rattling of straight cables on steel girder to girder restrainers. It is doubtful that it is useful elsewhere and, in some cases, may be harmful.

3. District traffic and resident engineers may recommend locations for deck or soffit access openings but Structure Design has the final say.

4. Jam nuts should be specified for cable restrainer end anchorages. Pressed sheet metal type lock nuts should not be allowed when jam nuts are specified on the contract plans.

5. A note "abrasive blastcleaning not required" should be placed on the plans in connections with concrete bolsters and other areas where bond is not important.

6. Holes in steel girder flanges may be drilled from the bottom up. This procedure obviates increasing the distance of the hole from the web to accommodate drilling equipment and permits proper edge distance on the flange.

7. Bearing plates of type C-1 restrainers should be fastened to the concrete. (Reference Bridge Standard Detail Sheet XS 12-57.)

8. The one inch studs should extend 10 inches beyond the ends of the swagged fittings at the ends of the ¾ inch cables. Longer studs should not be permitted in order to correct field errors.

9. At girder to pier locations, turnbuckle/cable assembly should be snugged, tightened one turn per 10 feet of cable length, adjusted for thermal movement, and finally secured with lock nuts.

10. Bolt holes in soffit cover plates should be 1 inch from edge of plate.

11. Details for cable drum units should permit an optional radius on the corners of the end plates.

12. Allow 4 feet of clearance for coring equipment (see Figure 28).

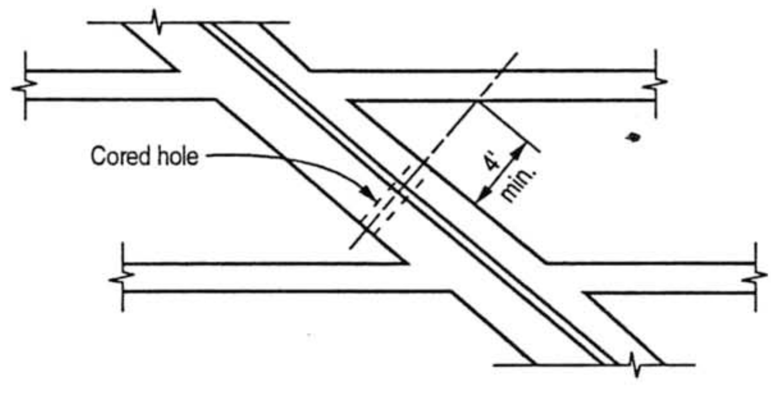


Figure 28

13. There should be a minimum of two restrainer units at each joint — preferably as far to each side and as symmetrical as possible.

14. The diameter of cored holes in concrete can be in ¼ inch increments up to 4 inches and 1 inch increments thereafter. Minimum distance from the center of a cored hole to adjacent face is 3 inches. Holes 6 inches in diameter and larger can be flush against a surface. The hole size versus number of cables must be investigated for each situation. Basically the hole must be large enough to contain all but one cable plus one swagged fitting; or contain all swagged fittings in the hole simultaneously.

15. Earthquake restrainers connecting the supported side of a hinge to the adjacent bent cap can be installed in box girder bridges without making access openings in the cantilever cells. However, there is a possibility that it may be necessary in some cases, or a contractor may prefer, that an access opening be made. Contract plans should indicate optional access openings unless there is a good reason why openings should not be made. See Figure 29.

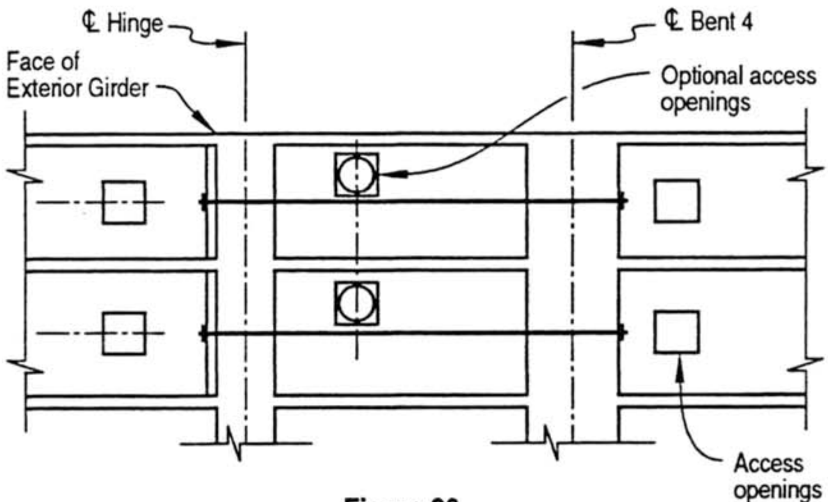


Figure 29

Optional openings should not be included in the estimate of quantities.

16. Pipes used for transverse restrainers should be 4 inch diameter double extra-strong filled with concrete. See Figure 37 for example design at the end of this memo.

17. Standard 8 inch diameter double extra-strong pipe hinge seat extenders have been developed. The pipes are capable of resisting a 100-kip vertical dead load reaction at an 8 inch extension. The seat extender pipe can also be used as a transverse key. An analysis of vertical and transverse loads at on expected extension must be performed. Generally long ductile restrainer cables are used in combination with the seat extender pipe.

18. Full penetration welds on bracket should be avoided as more economical designs using fillet welds are available.

19. Neoprene sleeves around cable bends protruding from cored holes are not required. Bends in cables shall have a curvature of not less than 4 inches. Edges of cored holes and fabricated devices shall be constructed to produce the proper radius.

20. Avoid details conflicting with maintenance inspection. An example of this is a supplemental support for a rocker type bearing where the support is designed in a manner which conflicts with inspection or future maintenance.

21. Example specifications, details and item costs are available from the Retrofit Specialist.

22. Welding details on restrainer brackets should provide for the sealing of all joined parts to prevent corrosion and eliminate galvanizing difficulties. Details equal to those in Figures 30 and 31 should be used for all connected steel components.

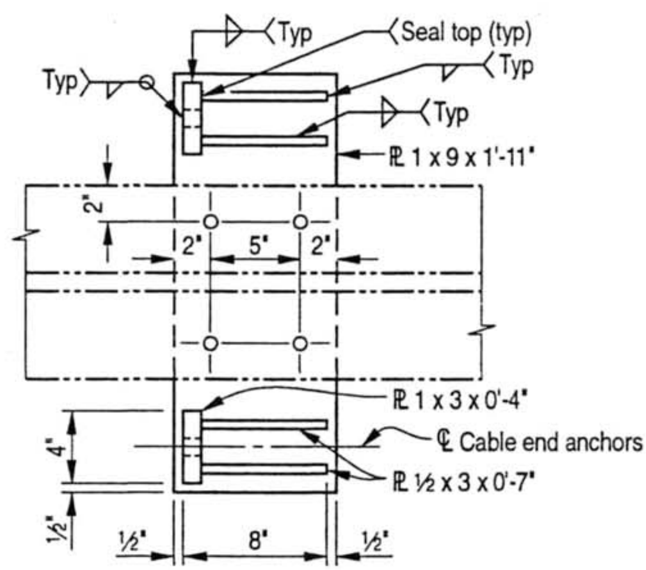
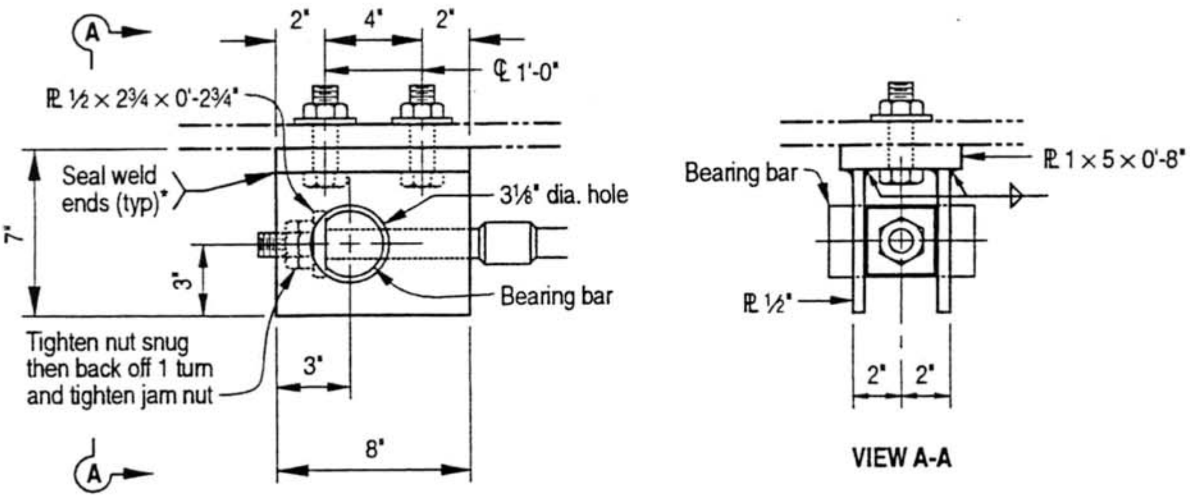


Figure 30



*Alternate: Extend 1 × 5 × 0'-8" plate and use fillet weld.

Figure 31

23. Provide a means of retarding nut removal by peening threads, spot welding, etc., on accessible units less than 12 feet above ground. Chemical thread coatings (i.e., Locktite) are not adequate.

24. The standard size turnbuckle comes in 1'-4" length, although other sizes are available with large orders.

Poor Details

Figure 32. Brackets are bolted to both sides of the girders and cables or rods connect the brackets to prevent the girders from moving apart longitudinally.

If restrainers on the two sides are tensioned differently or if one tendon fails, the eccentric loading can cause a bracket to rupture the girder web and the remaining tendons become ineffective prematurely. A ruptured web could cause the girder to fail in shear.

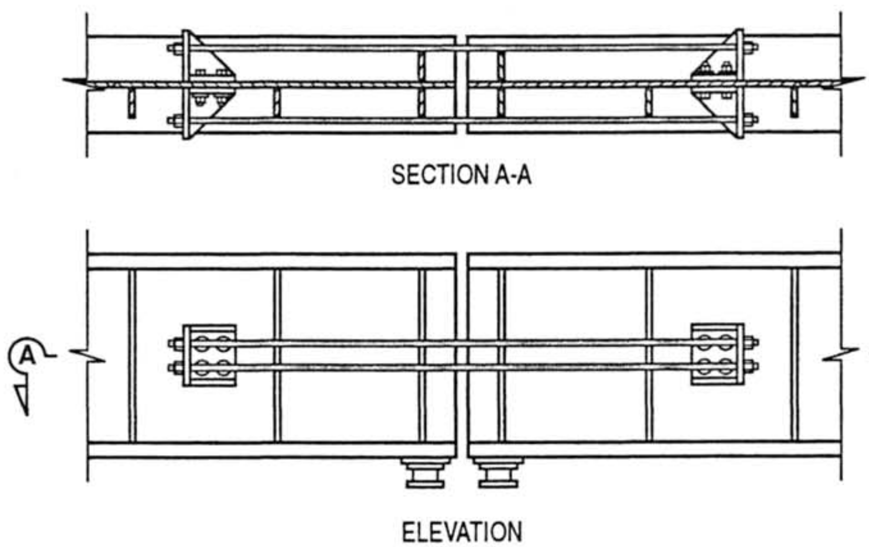


Figure 32

Figure 33. Supplemental supports scabbed onto the face of an abutment or pier face by means of dowels grouted in drilled holes should be cautiously selected as a retrofit solution. The magnitude of impact of the superstructure dropping even a slight distance, vertical seismic accelerations, horizontal friction, and "snagging" of remnants of the bearings on the supplemental support can tear the supplemental support from the face of abutment or pier. This detail can be an effective solution by allowing for movement in front of the sole plate and/or by extending the sole plate providing a continuous planar surface to slide on the support, and if all the forces noted above are addressed.

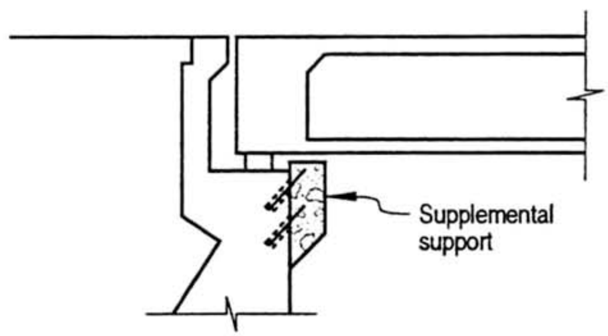
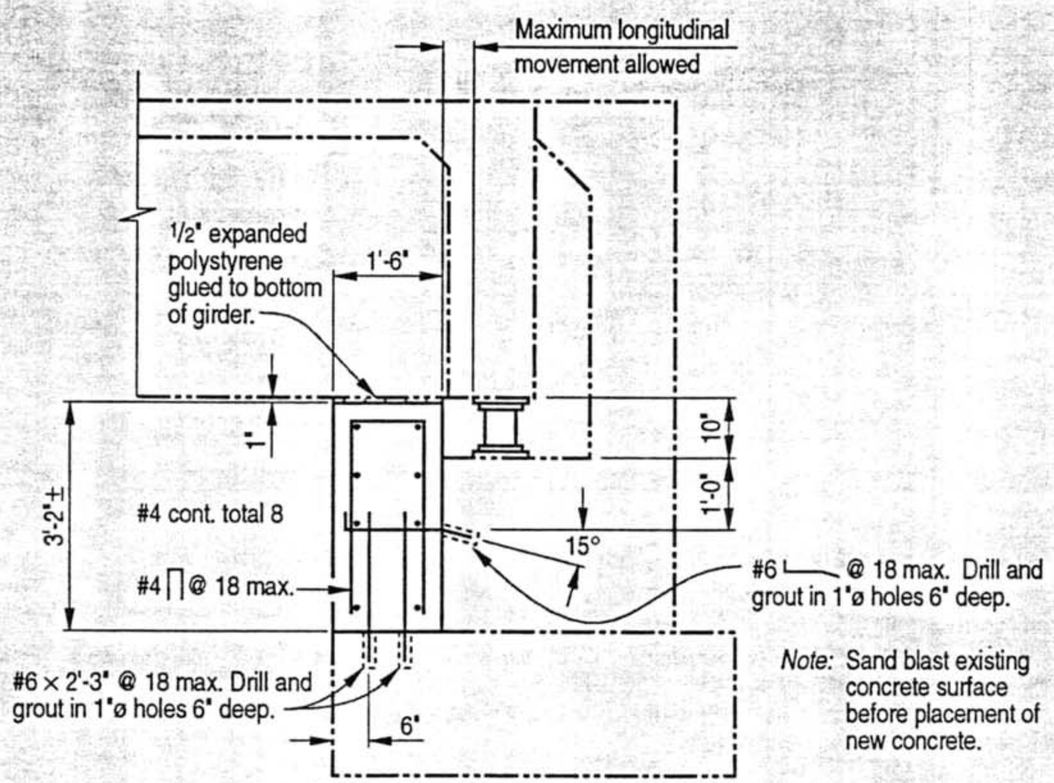
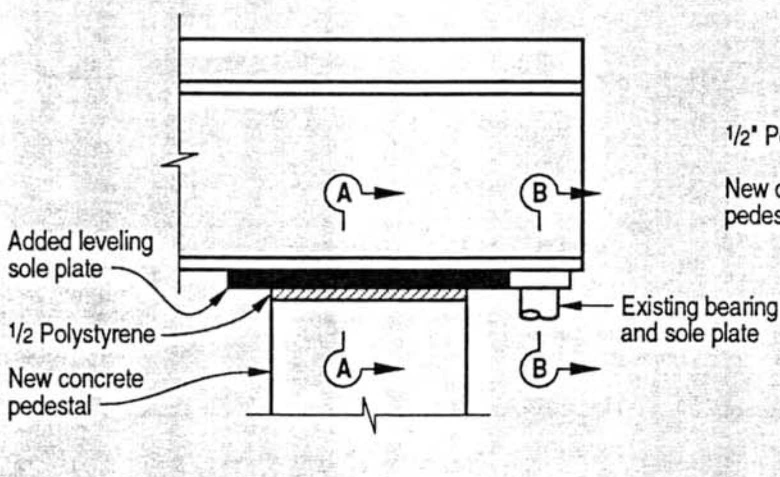


Figure 33

Exhibit A

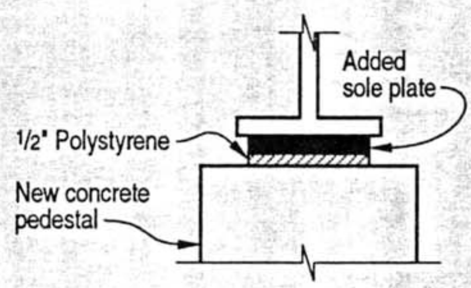


Typical Abutment Section
No Scale

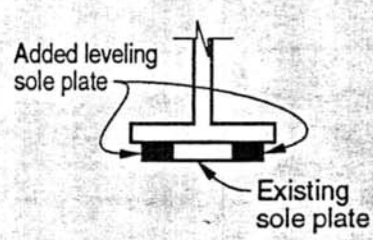


Note: Added leveling sole plate to be same thickness as existing sole plate.

Steel Girder Modification



Section A-A



Section B-B

Coring Existing Concrete

It is desirable that holes cored through concrete do not cut through reinforcing steel, hinge hardware or prestressing tendons. Although coring through steel is usually more of a construction nuisance than a structural problem, it is desirable to avoid it whenever possible. Designers should try to locate holes where they will not interfere with the above elements and take special care to avoid structurally critical reinforcement.

Coring through a few prestressed strands near the end of a precast-prestressed concrete girder may be unavoidable and is usually not structurally serious. However, special consideration should be given to locating holes to be cored in bridge members post-tensioned with rods or large multi-wire or multi-strand tendons.

Before locating holes to be cored in existing prestressed members, the designer should determine the method of prestressing by looking at the as-built microfilms, shop plan microfilms, construction final reports or construction photos. If it cannot be determined otherwise it should be assumed that prestressing was done by means of rods or large tendons.

In all cases where members prestressed by rods or large tendons are to be cored, the designer should notify the Specification Engineer by using form DPD-SD51. The Specification Engineer will include appropriate clauses in the Special Provisions.

Restrainer Materials Data

Three quarter inch cable, galvanized: See *Standard Specifications,* Section 75-1.035 – Bridge Joint Restrainer Units, for full description.

Minimum ultimate tensile breaking strength = 46 kips.

$A_S = 0.222 \text{ in}^2$

E = 14,000,000 psi (minimum specified before yielding)

E = 18,000,000 psi (after initial stretching)

Load Factor Design: Assume yield strength = 85% × 46 = 39.1 kips

High strength bars, galvanized: ASTM A7-22 with supplementary requirements (the supplementary requirements specify a minimum elongation of 7 percent in 10 bar diameters).

Diameter inches	Cross Sections Area Inches	Ultimate Strength ksi	Yield Strength ksi	Yield Strength kips
1*	0.85	150	120	102
1¼	1.25	150	120	150
1⅜ *	1.58	150	120	190

* Do not use these unless you have a verified source.

E = 30,000,000 psi

Galvanizing has promoted field problems related to the installation of high strength rods. Three types of rods are used - the Dywidag rod, K&M smooth rods, and Mukosil rods.

The Dywidag rods are galvanized after being threaded. Therefore the rod ends must be hot-brushed immediately after galvanizing. Even after this operation, placement of end nuts is difficult. K&M smooth rods are threaded after being galvanized. Then, after installation, the ends are coated with zinc-rich paint. Neither galvanizing nor threading of rods compromise the strength of either type of rod nor the anchorage requirement of 90% of the Minimum Ultimate Strength of the rod. (Reference UCLA Report.)

If any damage to the galvanizing occurs, zinc-rich paint must be applied to the affected area.

Another area of concern is that the standard locking devices are not effective on the Dywidag and Mukosil rods. Set bolts positioned properly must be applied to prevent lock nuts from vibrating off rods.

Rods longer than 30 feet should be avoided. Stock lengths are 30 feet and galvanizing tanks will not accommodate length greater than 30 feet.

Cable and Bar Tension Tests

The stress-strain curves shown in Figure 34 were obtained by tensioning specimens from near zero stress to specified minimum yield stress (0.85 times the minimum breaking strength for cables) for 14 cycles and then to failure.

The stress-strain curves shown in Figure 35 were made by tensioning bars and cables to failure but releasing the load to nearly zero at approximately one inch increments of stretching.

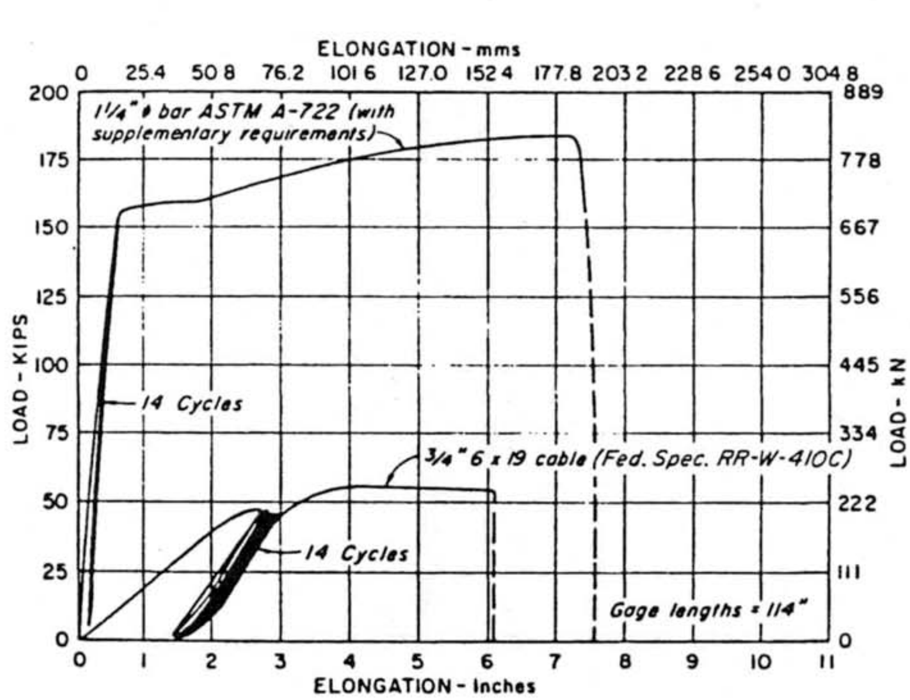


Figure 34

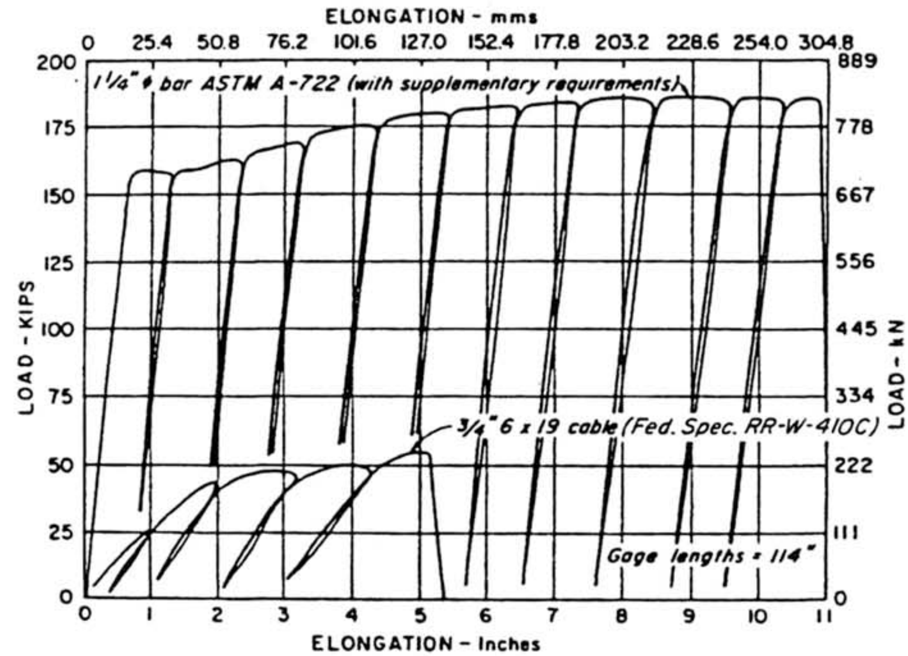

ELONGATION - mms
0 25.4 50.8 76.2 101.6 127.0 152.4 177.8 203.2 228.6 254.0 304.8
200 175 150 125 100 75 50 25 0
889 778 667 556 445 334 222 111 0
LOAD - KIPS
LOAD - kN
1 1/4" ϕ bar ASTM A-722 (with supplementary requirements)
3/4" 6 x 19 cable (Fed. Spec. RR-W-410C)
Gage lengths = 114"
0 1 2 3 4 5 6 7 8 9 10 11
ELONGATION - Inches

Figure 35

Design of Retrofitting Devices

Earthquake restrainer devices should fail in a ductile rather than brittle manner when subjected to ultimate loading. Therefore, the brackets, connections, and anchorages should be at least 25 percent stronger than the cables or rods.

Brackets and connections should be designed so that they will not fail if some of the restrainer cables or rods in the unit are misadjusted or fail prematurely.

The following ultimate strengths should be assumed for designing connections and determining the adequacy of supporting members:

¾ inch cables $F_m = 53$ kips.

1¼ inch H.S. rods $F_m = 188$ kips

(use $53 \times 1.25 = 66.2$ kips and $188 \times 1.25 = 235.0$ kips per cable and rod, respectively)

Bolted Connections shall be designed as a bearing type:

H.S. Bolts (A325)	Allowable Shear ($F_v = 0.6\ F_t$)	Allowable Tension ($F_t = \phi F_\mu$)
¾"	20.5 kips	34.1 kips
⅞"	28.3 kips	47.1 kips
1"	37.1 kips	61.8 kips
1⅛"	40.1 kips	68.1 kips

Combined Tension and Shear: $F_{vc} = \sqrt{(F_v)^2} - (0.6 f_t)^2$

where:

F_{vc} = Allowable shear per bolt for combined shear and tension
ϕ = Reduction Factor = 0.85
F_v = Allowable shear per bolt (kips)
f_t = Applied tension per bolt (kips)
F_μ = Ultimate tensile strength based on lab tests

The following allowable stresses should be used for designing ASTM A-36 steel brackets for ultimate conditions:

Tension or Compression	= 36,000 psi
Shear	= 26,000 psi
Bearing $\frac{LF_{\mu}}{1.18d}$ or 3.0 of F_{μ} whichever is smaller	
F_{μ}	= 58,000 psi
Groove welds	= 36,000 psi
Fillet welds	= 26,000 psi

Bearing plates for restrainer end anchorages should be sized to satisfy both bearing against the concrete surface and punching shear of the concrete member. The bearing resistance should be determined by the expression provided in the following treatise for pipe shear keys. A_g in that expression equals the net area excluding the cored hole. Punching shear plate size can be selected from the chart in Figure 36.

The pipe shear keys must satisfy shear and bearing criteria. The applied load is that resulting from a seismic analysis, whether a static or dynamic approach. The shear strength of the pipe is the value above, 26,000 psi, for ASTM A-36 brackets. The concrete filler is used to prevent collapse of the pipe and should not be given any value for shear resistance. The pipe must be contained in the cored hole by means of bearing resistance of the surrounding concrete. Each situation requires an analysis because of the variables due to joint skew, pipe length, and relationship of cored hole to pipe diameters. The designer must make some judgements of effective length and width of bearing areas. Refer to Figure 37. Allowable bearing resistance is determined by: $B = \phi \times 0.85 \times f'_c \times A_g$, where B = the calculated seismic shear force; $\phi = 0.9$ (seismic); f'_c = 4,000 psi (for existing structures); and A_g = effective bearing area.

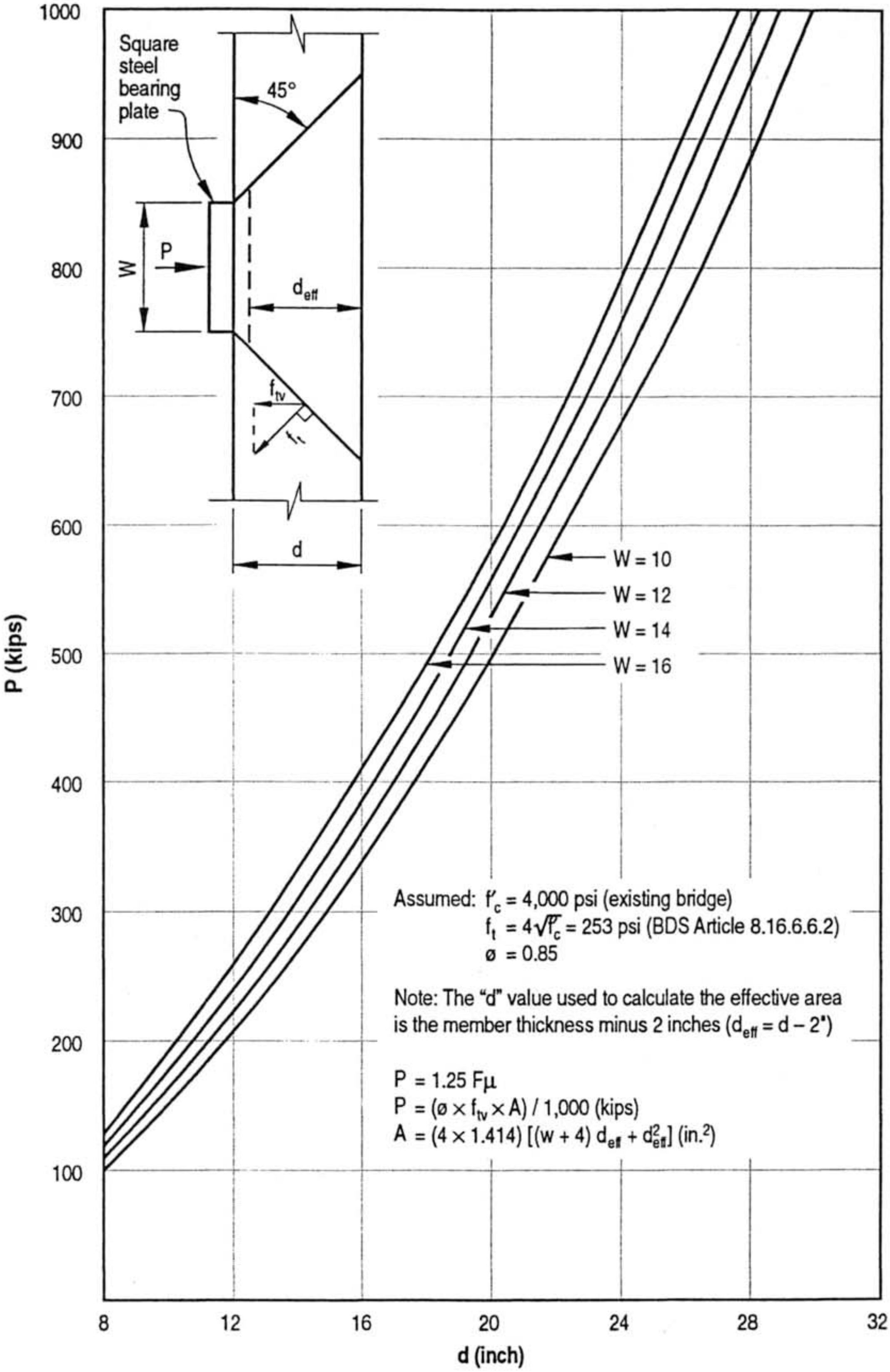


Figure 36. Resistance of Concrete Wall to Punching

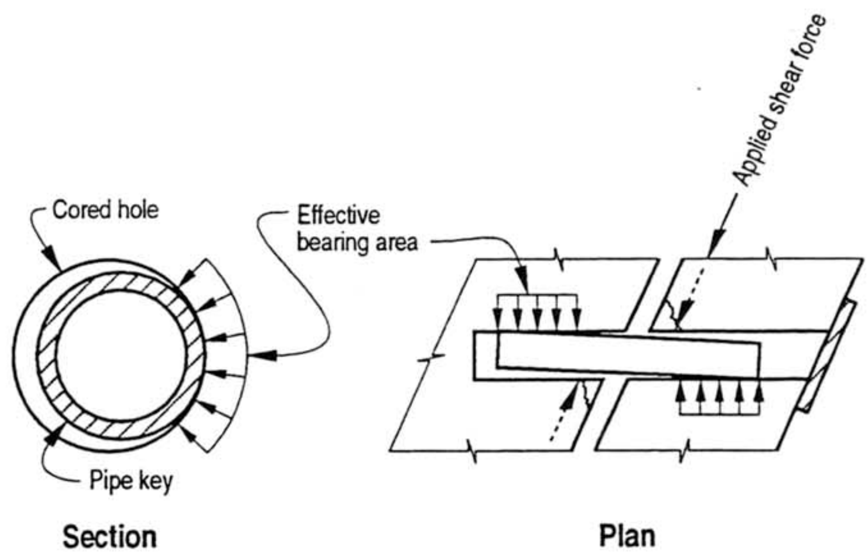


Figure 37

There are a few things that must be considered when using transverse pipe key, among which are:

a) The cored hole shall be as small as possible to fit the galvanized outer diameter of the pipe.

b) The acute corner of the cored hole at the expansion joint should not be considered effective in bearing in heavily skewed bridges.

c) The concrete filled pipes are very heavy and awkward to hoist into confined spaces. Consideration can be given to filling the pipes after they are positioned near the cored holes.

d) When unusually restricted space exists, longitudinal restrainer cables can utilize the transverse key holes by threading through the pipes, hence, reducing the number of cored holes. Provisions for allowing cable slippage through the concrete fill is necessary in these cases.

Floyd L. Mellon

Jerry A. McKee

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RJZ:jgf